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QUABBIN RESERVOIR — RECENT DEVELOPMENTS IN THE CONSTRUCTION OF THE WARE-SWIFT WATER SUPPLY FOR THE METROPOLITAN DISTRICT

BY FRANK E. WINSOR, MEMBER *

(Presented at a meeting of the Boston Society of Civil Engineers held on May 18, 1938)

SYNOPSIS

THIS paper is aimed to supplement the complete and valuable paper entitled "Boston Metropolitan Water Supply Extension," by Karl R. Kennison, Assistant Chief Engineer, Metropolitan District Water Supply Commission, which was published in the "Journal of the New England Water Works Association," Volume XLVIII, No. 2, 1934, page 147. Many other papers have been published from time to time, as shown in the accompanying bibliography, particular attention being directed to the publications of this society, the American Society of Civil Engineers, the New England Water Works Association and various technical journals.

HISTORY

In the year 1895, fourteen cities and towns located within 10 miles of the State House in Boston were constituted by legislation (chapter 488, Acts of 1895) a Metropolitan Water District. This act provided that any other city or town so located could upon application become

* Chief Engineer, Metropolitan District Water Supply Commission, 20 Somerset Street, Boston, Mass.

a member of said District. At present twenty cities and towns with a combined population of 1,600,000 are members of the District, and there are fifteen potential members with a combined population of over 400,000. The city of Boston was paid by the Metropolitan Water District about \$14,000,000 for reservoirs in the Sudbury system, Lake Cochituate, Chestnut Hill Reservoir, Sudbury and Cochituate aqueducts, and the Mystic works at Mystic Lake. Other cities and towns were paid smaller amounts, aggregating about \$1,300,000, for works needed in the development of the Metropolitan system.

The principal works built by the Metropolitan Water Board and its successors in title, the Metropolitan Water and Sewerage Board and the Metropolitan District Commission, and mainly completed in the early years from 1895 to 1905, were: Wachusett Reservoir, with dam, at Clinton; the Wachusett Aqueduct, to carry water to the Sudbury Reservoir; the Sudbury Reservoir, begun by the city of Boston; the Weston Aqueduct, from Sudbury Reservoir to a point in the town of Weston on the opposite side of the Charles River from Norumbega Park in Newton; Spot Pond, in the northerly part of the District, which was rebuilt, pipe lines to connect the system with the various members of the District; and several high service reservoirs, standpipes and pumping stations. Spot Pond and Chestnut Hill reservoirs became the main distributing centers. The distribution system has been extended from time to time as required.

The cost of the above work, including the amounts paid to the city of Boston and other communities, aggregating about \$15,300,000, but not including any expenditures for the Ware-Swift Water Supply, is about \$50,000,000.

Following exhaustive investigations, description of which may be found elsewhere, the Legislature, by chapter 375, Acts of 1926, established the Metropolitan District Water Supply Commission as a construction commission, and under this chapter and chapter 321, Acts of 1927, authorized the extension of the water supply system westerly to the Ware and Swift rivers, and directed that the work be prosecuted forthwith. Chapter 111, Acts of 1927, also required the commission to connect the Hopkinton and Whitehall reservoirs in the Sudbury system to the Sudbury Reservoir, and the Ashland Reservoir to the Sudbury Aqueduct, which work was practically completed in 1928 at a cost of \$638,534.

The organization of the engineering staff began on October 1, 1926, and the construction of shafts on the tunnel from the Wachusett Reservoir to the Ware River began the following March.

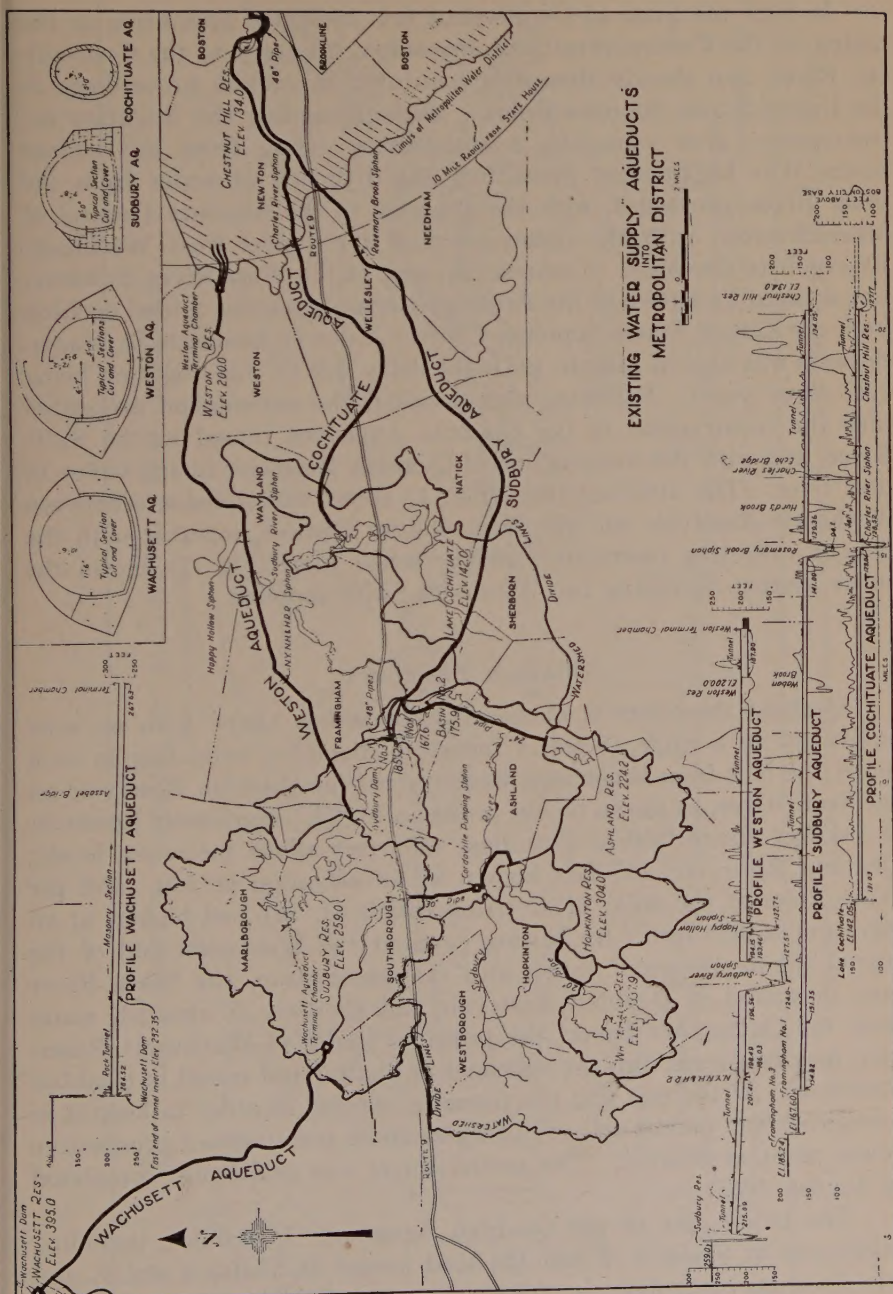


FIG. 1.— EXISTING WATER SUPPLY AQUEDUCTS FROM WACHUSETT RESERVOIR TO THE METROPOLITAN DISTRICT

In 1927 the State of Connecticut first indicated opposition to the taking by the Commonwealth of any water tributary to the Connecticut River, and shortly thereafter instituted injunction proceedings in the United States Supreme Court. Notwithstanding this litigation the commission, after obtaining a satisfactory ruling from the United States War Department permitting the diversion of water from the Ware River, proceeded, with the approval of the Attorney General of Massachusetts, with the construction of the tunnel from Wachusett Reservoir to the Ware. Construction west of the Ware River, however, was delayed by advice of the Attorney General, pending a final decision from the United States Supreme Court. A decree favorable to Massachusetts was filed in March, 1931, the delay due to this litigation being over three years. Following this decision, the commission proceeded with the construction of the Quabbin Aqueduct tunnel to the Swift River, and with the construction of Quabbin Reservoir in the valley of that river. The attached plan (Fig. 1) shows the aqueducts between Wachusett Reservoir and the Metropolitan District, together with the present collecting reservoirs; and attached plan (Fig. 2) shows the entire system, including the Ware and Swift supplies.

QUABBIN AQUEDUCT

Quabbin Aqueduct is a tunnel of horseshoe shape with an area equivalent to a circle 12 feet 9 inches in diameter, extending in rock under the hills from Wachusett Reservoir to the Quabbin Reservoir on the Swift River, as shown on the profile in Fig. 2. Wachusett Reservoir is 395 feet above Boston City Base (389 feet above mean sea level); Quabbin Reservoir, 135 feet higher, at an elevation of 530; and the Ware River at the point of diversion (Shaft 8), 261 feet higher, at an elevation of 656 feet. By closing gates at the Wachusett end of the tunnel (Shaft 1), water taken into the tunnel from the Ware River may be stored in Quabbin Reservoir and similarly, if desired, water from the Quinapoxet River, the principal feeder of Wachusett Reservoir, may be turned into the aqueduct at Shaft 2 and stored in Quabbin Reservoir. At the east end the tunnel is lowered in order to keep it in sound rock and permit delivery of water under pressure to a prospective power plant at Shaft 1. The entire tunnel was practically completed in August, 1935.

The entire cost of the Quabbin Aqueduct, 24.6 miles, including gatehouses at Shafts 1, 8 and 12, head houses at Shafts 4 and 9, and miscellaneous equipment, is: for construction, \$14,601,615; real estate,

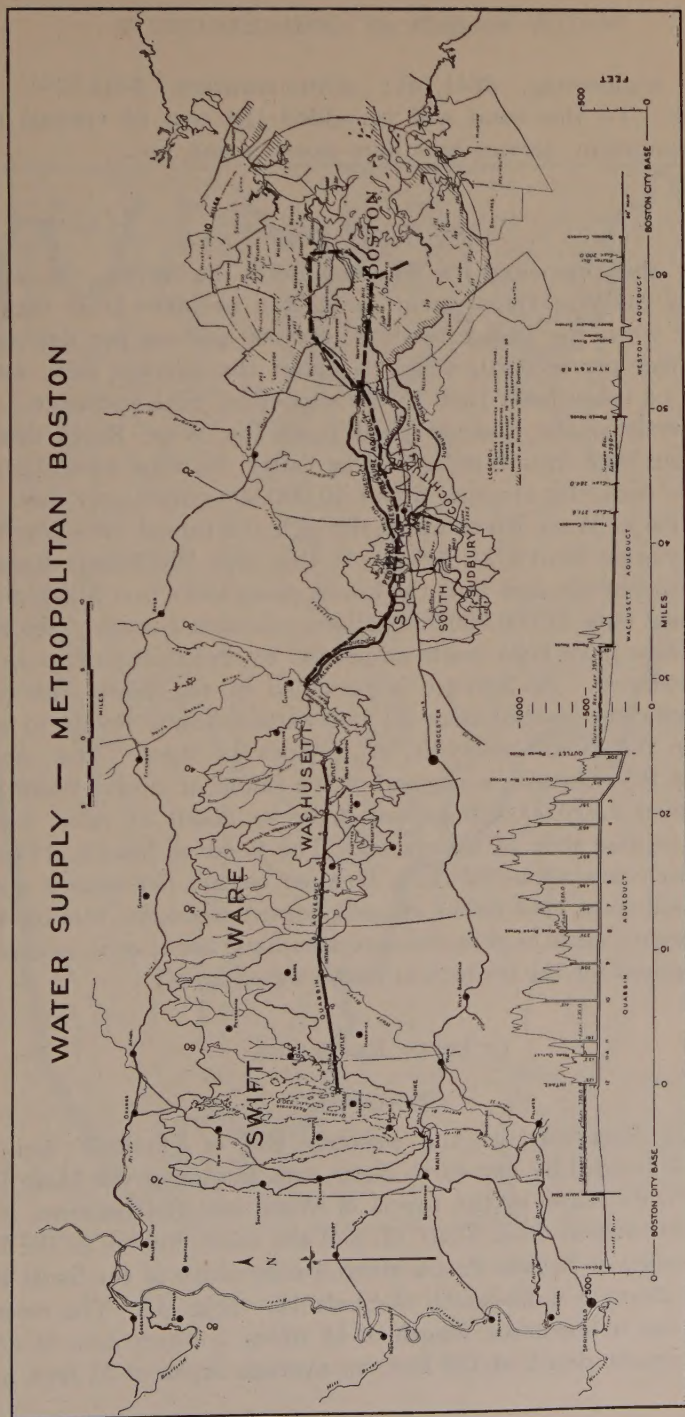


FIG. 2. — ENTIRE WATER SUPPLY SYSTEM FOR METROPOLITAN BOSTON, INCLUDING THE WARE AND SWIFT SUPPLIES

\$189,963; engineering, \$841,161; administration, \$444,529; total, \$16,077,268. To this total will be added the cost of various minor items of equipment, service buildings, power plant, etc.

WARE RIVER SUPPLY

Under the provisions of Massachusetts legislation, as slightly modified by the War Department, water can be taken from the Ware River only when the flow exceeds 85,000,000 gallons per day (about 131 cubic feet per second), which is about the average flow, and no water may be taken from June 15 to October 14, both inclusive. With these restrictions, the average yield from the Ware River drainage area of about 96.8 square miles will be (after the time when Quabbin Reservoir is ready for storage) about 40,000,000 gallons per day. The first diversion of Ware River water through the tunnel into Wachusett Reservoir was on March 21, 1931. At this time Wachusett Reservoir was 45 feet below the flow line, the lowest point since first filled in 1908, and contained only 11,000,000,000 gallons available storage. Since that time diversions have been made as needed to replenish storage in Wachusett Reservoir. No storage is provided in the Ware, the tunnel being of sufficient size to take all but extraordinary floods to either Quabbin or Wachusett reservoir, or to both of them. The capacity of the Ware River intake works and of the tunnel to divert Ware River water is about 1,910,000,000 gallons daily, or about 31 cubic feet per second per square mile of watershed. The flood of March, 1936, the peak of which was about 62 cubic feet per second per square mile, is the only flood since 1913 (when records began) to exceed this capacity. About 20 square miles of this drainage area have been, or it is expected will be, acquired for the protection of the water supply.

QUABBIN RESERVOIR

Description

Quabbin Reservoir, at the westerly end of Quabbin Aqueduct (Shaft 12), is formed by two earth dams, one, known as the Main Dam, across the Swift River in the towns of Ware and Belchertown, about $7\frac{1}{2}$ miles downstream from Shaft 12, and the other, known as the Dike, across the valley of Beaver Brook about 1 mile south of the Swift River divide, and about 5 miles south of Shaft 12. (Fig. 3.) The reservoir when filled has a maximum length of 18 miles, a water area of 25,200 acres, a maximum depth of 150 feet, an average depth of 51 feet, and a



FIG. 3.—QUABBIN RESERVOIR

capacity of about 415,000,000,000 gallons, or about 1,280,000 acre feet. The flow line is at elevation 530 and is 177 miles in length, including the flow line around 110 islands aggregating 30 miles. The only shallow flowage of consequence is at the extreme upper ends of the east and middle branches of the reservoir, where shallow flowage dams will be built to keep these areas flooded at all times.

This reservoir has caused the elimination of the four towns of Enfield, Greenwich, Prescott and Dana, the annexation of these towns to contiguous towns, and the readjustment of the lines of the three counties of Worcester, Hampshire and Franklin, all of which changes became effective as of April 28, 1938. The reservoir flooded about 16 miles of the Athol branch of the Boston & Albany Railroad, and has resulted in the abandonment of this branch between Belchertown and Athol, a distance of 28.5 miles. A 66,000-volt and a 22,000-volt transmission line of the New England Power Company which crossed two branches of the reservoir have been relocated outside the reservoir.

Land

Of the 185.9 square miles of the Swift River drainage area tributary to Quabbin Reservoir, about two thirds, including the area to be flooded, have been acquired for the protection of the water supply, as shown in Fig. 4. The area thus owned includes all islands and peninsulas and a marginal strip one half mile or more in width. The marginal strip in the portions of the reservoir which are within $1\frac{1}{2}$ to 2 miles of the watershed limit extends generally to the divide. In addition to land owned by the commission, there are about 6.5 square miles of this watershed in public or semi-public ownership which insures reasonable sanitary protection.

Circulation in Reservoir

In order to secure acceptable circulation of water entering and leaving the reservoir, baffles were built north of Shaft 12 connecting two islands, one half mile and 4 miles in length, respectively, with the mainland, as shown in Fig. 3. These baffles are formed by earth embankments aggregating about 2,200 feet in length, with a maximum height of about 100 feet and a volume of 500,000 cubic yards. The baffles increase the distance through which water entering the reservoir through the east branch of the Swift River (the principal feeder of Quabbin Reservoir) and from the Quabbin Aqueduct at Shaft 11A (Ware River water and Quinapoxet River water if diverted into the Aqueduct) must

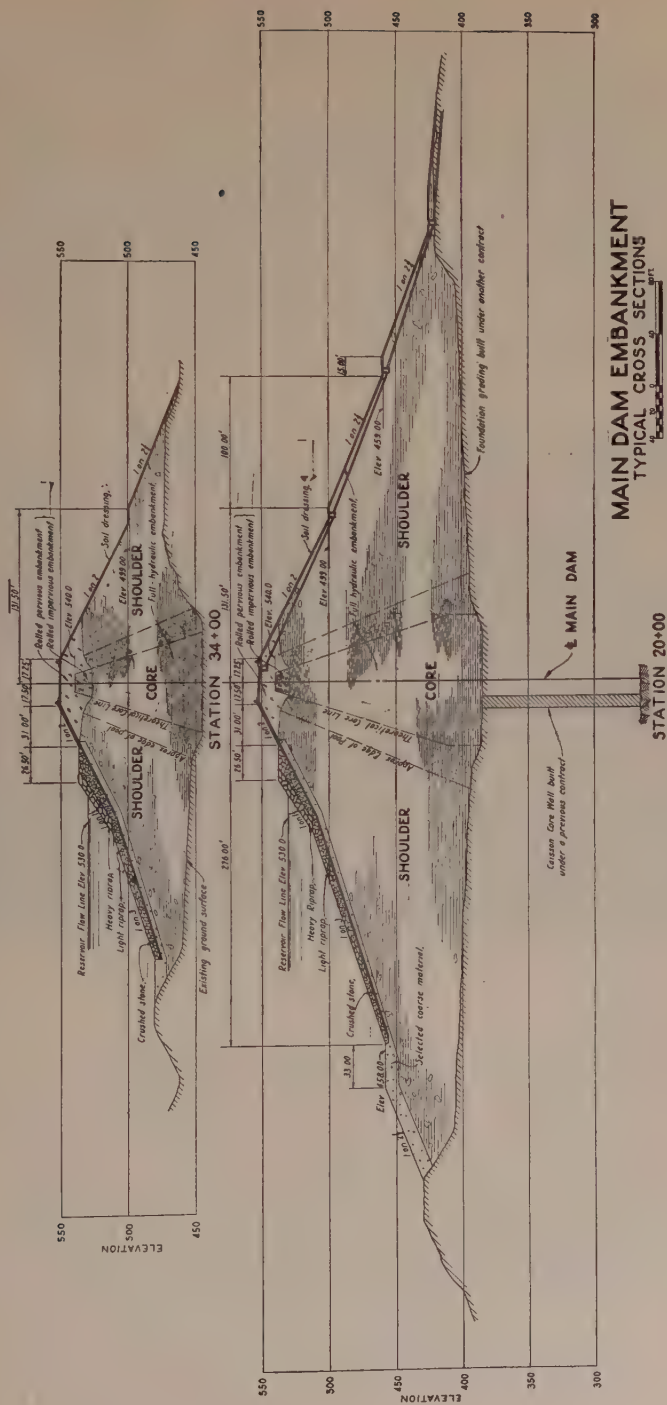


FIG. 4. — QUABBIN DAM — CROSS SECTIONS

travel to reach the out-take (Shaft 12) by about 10 miles, mainly through deep and wide portions of the reservoir.

The benefits of long-time storage in the reservoir are secured in an unusual degree. Water which originates on privately owned land must travel one half mile or more through land of the Commonwealth to reach the reservoir, after which it must travel a minimum distance of about 6 miles through the deep and wide portion of the reservoir to reach the out-take at Shaft 12. Of the water falling on privately owned land, about 67 square miles in area, that from the east branch of the Swift River, 35 square miles, must flow 12 to 13 miles through the reser-



FIG. 5. — LOOKING NORTHWESTERLY ALONG THE EAST BRANCH BAFFLE

voir to reach Shaft 12; water from the middle branch, 20 square miles, must flow 6 to 11 miles through the reservoir; and water from the west branch, about 12 square miles, must flow 14 to 16 miles through the reservoir. The weighted average distance of travel through the reservoir of water originating on privately owned land is 12 miles. All highways will be well back from the flow line, at only four points being less than one half mile distant, and in these four cases being, respectively, 8, 11, 13 and 15 miles from the intake at Shaft 12. No drainage from highways enters the reservoir less than 8 miles from the intake at Shaft 12.

Water stored in the reservoir from the Ware River must travel a

minimum distance of 4 miles on the Ware Watershed to reach the intake at Shaft 8, thence 9 miles westerly in the Quabbin tunnel to Shaft 11A, thence about 10½ miles through the reservoir to Shaft 12.

Water leaving Shaft 12 for Wachusett Reservoir direct will travel 25 miles to Wachusett Reservoir and 7 miles through this reservoir to Wachusett Dam, where it will enter Wachusett Aqueduct on its way to the Metropolitan District. Wachusett Reservoir has a storage capacity of 65,000,000,000 gallons and an average depth of 50 feet.

Relocation of Roads

Thirty-four miles of state highway and 2 miles of county highway have been built in substitution for 242 miles of highway to be abandoned. The average cost per mile of state highway (exclusive of engineering) is \$42,600, and the Department of Public Works reimbursed the commission for the bituminous macadam surface and guard rails at an average amount of \$16,150 per mile.

Cemeteries

About 8,000 bodies from eighteen community cemeteries and other smaller cemeteries have been or will be removed from the area acquired by the Commonwealth. Most of these removals are to the new Quabbin Park Cemetery, located in the town of Ware, outside the drainage area of Quabbin Reservoir, contiguous to the main state road (Route 9) between Ware and Belchertown, and about 2 miles easterly of the Main Dam. This cemetery was established in 1932, following the decision of the Supreme Court in the Connecticut diversion suit. To June 30, 1938, about 4,300 bodies had been removed to Quabbin Park Cemetery at an average cost of about \$13.50 per body, and 935 bodies to outside cemeteries at about \$31 per body. The cost of construction and maintenance of Quabbin Park Cemetery has been about \$85,000, against which can be credited receipts from sales of lots, interment fees, etc., totalling about \$2,500.

Clearing

The area to be flooded will be cleared of trees, vegetation, buildings, etc., and sources of pollution, such as privies and barnyards, will be disinfected. In 1936, portions of the area to be flooded were partially cleared as an unemployment relief project.

Reforestation

The general reforestation of such areas above the flow line as are clear or partially clear is under way both at Quabbin Reservoir and in areas acquired on the Ware Watershed. Nurseries have been established to provide stock for reforestation; and for the early planting, nursery stock was purchased. Conifers, such as red and white pine, various varieties of spruce, larch and some hemlock are being most commonly used. Some hardwood may be used.

QUABBIN DAM

The Main Dam of Quabbin Reservoir is now being built across the valley of the Swift River in the towns of Ware and Belchertown and in the former town of Enfield. It is an earth embankment containing about 4,000,000 cubic yards, over 2,000,000 cubic yards having been placed to date. The dam is 2,640 feet long, 170 feet high above the river level, 295 feet high above the lowest bedrock in the foundations, and with its top 20 feet above the flow line of the reservoir. The cross section and type of construction are shown in Fig. 4. The structure is to be connected with the state highway (Route 9) between Belchertown and Ware by an access road over the top of the Dam and past the site of the administration buildings.

Work at the Dam began with the construction of stream control works. These works consist of a diversion conduit, mainly a rock tunnel under the westerly abutment of the Dam, connected to the river by upstream and downstream channels. This conduit is concrete lined, horseshoe shaped, about 1,200 feet long, with a normal inside cross-sectional area equivalent to a circle 30 feet in diameter. The control works were built by the Northern States Contracting Company between August, 1931, and August, 1933. The contract cost was \$544,610.60.

The Swift River Valley at the site of the Dam is filled with porous, glacial drift, and a concrete cut-off wall was built by sinking forty reinforced concrete caissons, all except one (the first exploratory caisson) being 45 feet long and 9 feet wide. The sinking of this core wall was facilitated by the lowering of the ground water at the site through extensive pumping operations, pumps being installed in some of the exploratory caissons to accomplish this lowering before the foundation contract was let. The caissons were sealed to the rock foundations, the rock foundations grouted, and the spaces between caissons filled with concrete.

QUABBIN DAM AND DIKE — CAISSON CORE WALLS

Caisson Sinking in Linear Feet, showing the Effect of Pumping upon the Air Pressures Actually Used

AIR PRESSURE	LINEAR FEET OF EXCAVATION						RULES AND REGULATIONS, MASSACHUSETTS DEPARTMENT OF LABOR AND INDUSTRIES HOURS OF WORK AND REST FOR EACH 24-HOUR PERIOD			
	ACTUAL CONDITIONS OF GROUND WATER AFTER LOWERING BY PUMPING		GROUND WATER AT ORIGINAL ELEVATION, NO PUMPING		GROUND WATER AT BOTTOM OF TRENCH, EXCEPT WHEN BOTTOM OF TRENCH IS ABOVE ORIGINAL GROUND-WATER ELEVATION		MAXIMUM TOTAL IN- WORK	MAXIMUM FIRST SHIFT UNDER COM- PRESSED AIR	MINIMUM REST IN- TERVAL IN OPEN AIR	MAXIMUM SECOND SHIFT UNDER COM- PRESSED AIR
	Dam	Dike	Dam	Dike	Dam	Dike				
Free air	1,930	1,448	512	46	512	46	—	—	—	—
0-10 pounds	283	400	757	692	757	680	8	4	$\frac{1}{2}$	4
10-18 pounds	100	177	481	523	460	517	8	4	$\frac{1}{2}$	4
18-26 pounds	56	38	351	481	326	431	6	3	1	3
26-33 pounds	4	0	241	381	212	286	4	2	2	2
33-38 pounds	—	—	139	217	79	91	3	$1\frac{1}{2}$	3	$1\frac{1}{2}$
38-43 pounds	—	—	112	183	32	12	2	1	4	1
43-48 pounds	—	—	64	160	—	—	$1\frac{1}{2}$	$\frac{3}{4}$	5	$\frac{3}{4}$
48-50 pounds	—	—	28	41	—	—	1	$\frac{1}{2}$	6	$\frac{1}{2}$
Over 50 pounds	—	—	—	64	—	—	—	—	—	—
Total excavation (feet)	2,373	2,063	2,685	2,788	2,378	2,063	—	—	—	—

Prior to sinking the caissons an open trench excavation was made across the valley to a depth of about 30 feet. This trench was subsequently filled with topsoil, consolidated by sheep's foot rollers in 6-inch layers, the caisson core wall extending into this refill a minimum of 25 feet.

Upstream and downstream cofferdams, to protect the work from floods during the foundation and embankment construction, were built of the materials excavated from the stream control channels and from the trench excavation for the core wall. These cofferdams were placed so as to form the toes of the embankment of the Dam. Three of the caissons of the core wall were sunk as exploratory caissons, and pumping from them demonstrated to prospective bidders the feasibility of lowering the ground water, thus resulting in substantial savings in the foundation costs.

The foundation work was done under contract with The Two Companies, Inc. The total cost of the contract, including \$265,929.95 spent for roads, spillway channel excavation, borrow pit investigations for the embankment construction, etc., was about \$1,606,629.88. In addition, exploratory work, including the three caissons, pumping, etc., cost about \$300,000. This work was begun in May, 1935, and was completed in the early summer of 1936.

The contract for the construction of the embankment was executed on August 24, 1936, with the Benjamin Foster Company, at its bid price of \$2,317,445. This work, for the remainder of the 1936 season, consisted mainly in the construction of the abutments and the foundations for the sluicing of the embankments. Work on the hydraulic fill started in the 1937 season with the filling of the pool on May 10, and between that time and November 11, when the contractor stopped hydraulic work for the season, about 2,000,000 cubic yards of embankment had been so placed. Between June 1 and November 11 an average of 14,000 cubic yards were placed per day, the maximum being 22,700 cubic yards on August 12. Operations were carried on generally with three 8-hour shifts, the average plant operation being about 21 hours per day. The pool was maintained by pumping water from the river through a 16-inch make-up water line. A barge floating in the pool carried four 12-inch, 315-horsepower pumps, which pumped into a 28-inch steel clear water line to the hog box located at the top of the hill forming the east abutment of the Dam. The hog box is about 100 feet above the finished top of the Dam. The embankment materials were borrowed upstream from the Dam and easterly of the river. Materials are conveyed on belt conveyors to the hog box.

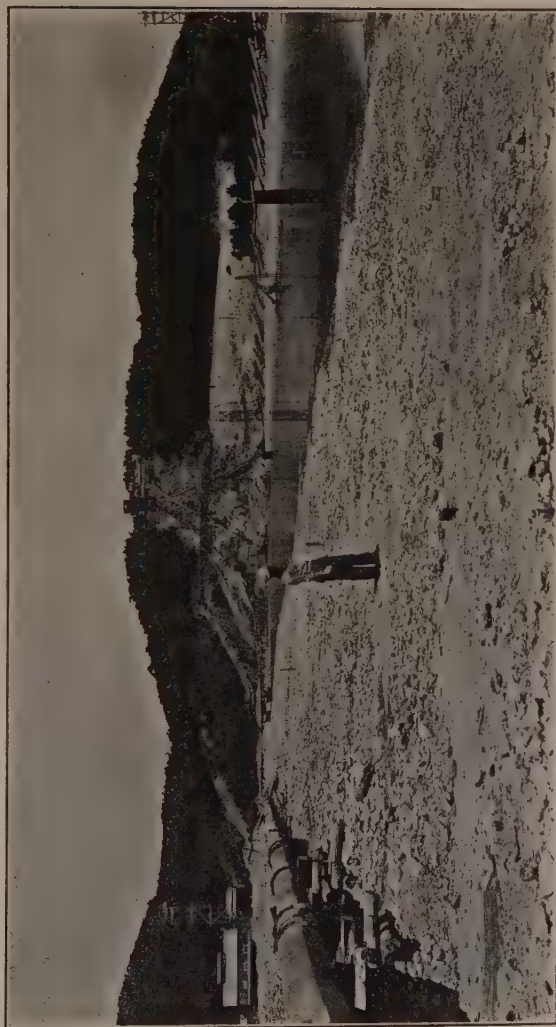


FIG. 6. — CONSTRUCTION AT QUABBIN DAM, SHOWING BEACHES, CORE POOL, AND HOG BOX, DREDGE PUMP AND SLUICING LINES ON HILL AT EASTERLY END

During 1937, two 2-cubic-yard Diesel power shovels loaded the materials into hoppers mounted on rails over 36-inch movable field belt conveyors. The field belt conveyors were moved by the power shovels which excavated in the pit. In addition to the power shovels, six scrapers (four 10-cubic-yard LeTourneau, one 7-cubic-yard Gar Wood and one 5-cubic-yard Gar Wood), hauled by six 75 to 90 horsepower Diesel tractors, excavated from other parts of the pit and deposited the material in a scraper dry box which fed the material to the belt conveyors. Materials were also excavated from a pit northeasterly of the hog box by two 2-cubic-yard Diesel power shovels loading into eight 5-ton trucks, the material being transported to the dry box and sent through a hopper to the main belt conveyor. The main conveyor up the hillside was 42 inches in width in three sections, each section being operated by 150-horsepower motors. After delivery to the hog box, mixing was accomplished by forcing water through a series of jets and allowing the material to flow through these jets into a trough which sloped steeply towards the sluicing lines. The mixture was carried through these lines by gravity on to the beaches. In the spring of 1938, a booster pump was installed in the delivery line to give greater velocity of discharge than could be obtained by gravity. The delivery lines are 20-inch steel pipe lined, in general, with rubber about one-quarter inch thick. The delivery lines are dismantled by adding or taking off one pipe at a time without shutting down the pumps. A 1-cubic-yard drag line operated on each beach to throw up the levees and to handle the pipe in the delivery line. The flow of materials over the beaches was regulated by baffle boards and raking. The quality of the core was continuously inspected by soundings with a rod and by sampling for inspection and laboratory analysis. Coarse intrusions were jetted. The jetting rigs consisted of a gasoline-driven pump of 500 gallons per minute capacity mounted on a float with a 2-inch jet pipe suspended from a head frame on the float. Cobbles and boulders from the borrow areas, which were rejected as too large to be delivered through the hog box, were placed in piles and later delivered by truck to be used as riprap on the upstream face of the Dam.

In addition to the embankment work, the contractor carried on earth and rock excavation in the spillway channel to be built through a natural ravine easterly of the Dam, the rock being used in the riprap for the upstream face of the Dam.

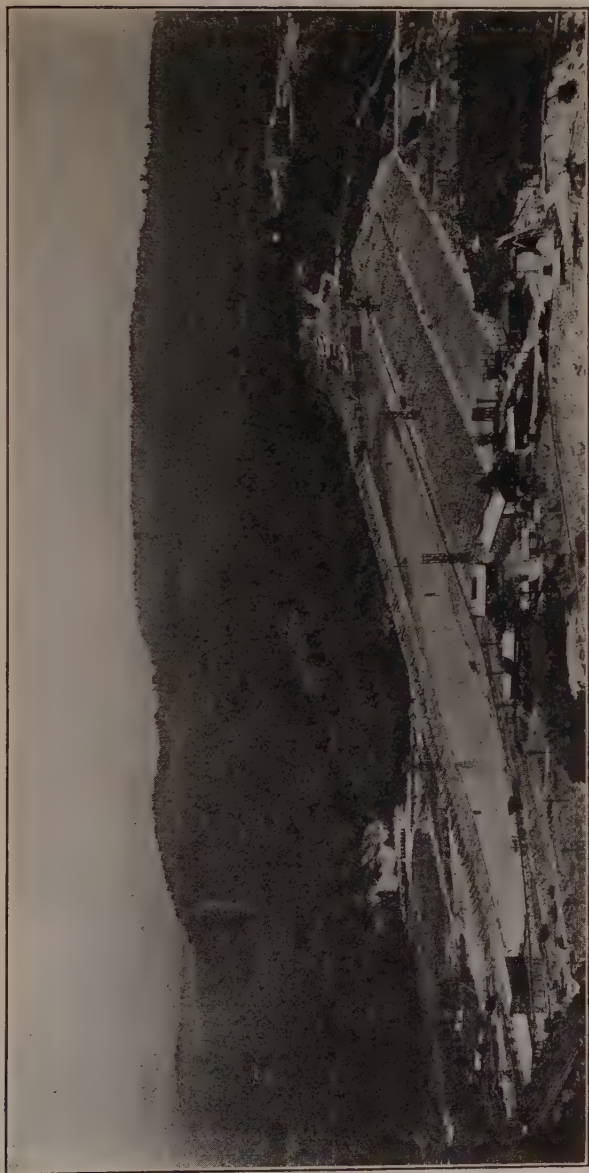


FIG. 7. — GENERAL VIEW OF CONSTRUCTION OF QUABBIN DAM, LOOKING SOUTHWESTERLY — ADMINISTRATION BUILDINGS SHOWN AT THE EXTREME RIGHT

Administration Buildings

The administration buildings, which will serve as general headquarters for maintenance of the Quabbin Reservoir and its watershed, are under construction at the westerly end of the Main Dam on the southerly shore of the west branch of Quabbin Reservoir, with a 10-mile view up that branch. The group consists of an office building, two residences, two garages for housing eighteen cars and trucks, an underground 60-foot by 70-foot hydroplane hangar, with connecting boat-house and general machine shop. The entire group is of fireproof construction protected by automatic sprinklers and heated by a steam plant in the basement of the office building equipped to burn either No. 5 oil or soft coal. Electric power is furnished by the Central Massachusetts Electric Company at 2,300 volts, and transformed and distributed from an underground vault to this group and to the Dam gate house. Auxiliary power for emergency operation is supplied from a 10 k. v. a. gasoline-driven generator set. The water supply is from a driven well, with 50,000-gallon storage tank on the hill to the southeast. The hydroplanes will, it is expected, be used for police patrol of the reservoirs and properties of the Metropolitan water system. Provision will be made for a short-wave radio receiving and broadcasting station. The contract was executed with Rich Brothers Construction Company, Inc., on July 28, 1937, for the construction of the administration buildings at a bid price of \$362,445. On June 10, 1938, this work was about 70 per cent completed.

QUABBIN DIKE

The Dike is about $2\frac{1}{2}$ miles northeasterly of the Main Dam on the former line which divided the towns of Ware and Enfield, and crosses the valley of Beaver Brook at about 1 mile southerly of the Swift River divide. The Dike is 2,140 feet long, 135 feet high above the former brook bed, 264 feet high above the lowest bedrock in the foundations, and with its top 20 feet above the flow line of the reservoir. The width at the top of the Dike is 35 feet, and the upstream slope, which is protected against wave wash with riprap, crushed stone and selected coarse material, has an average slope of about 1 on 3. The downstream slope is covered with topsoil, seeded and grassed. In this face is built a tile drainage system for preventing erosion of the slopes from heavy rains. Also there are two drainage berms. This face has a slope of 1 on 2 for the upper 50 feet and 1 on $2\frac{1}{2}$ for the lower part of the Dike. Stone



FIG. 8.—QUABBIN DIKE — DOWNSTREAM FACE LOOKING NORTHERLY FROM ACCESS ROAD

walls are built on either side of the roadway extending across the Dike. This structure is almost identical in cross section and type of construction with the Main Dam, now under construction and heretofore described. Access to the Dike is over a part of the present road (to be later abandoned as a public road) to Enfield Village from U. S. Route 9, and over macadam roads extending both over the top of and on the downstream side of the Dike.

The Dike was built under two main contracts, one for the foundations and one for the embankment. Explorations by means of bore holes and by means of sinking an exploratory caisson at the site indi-

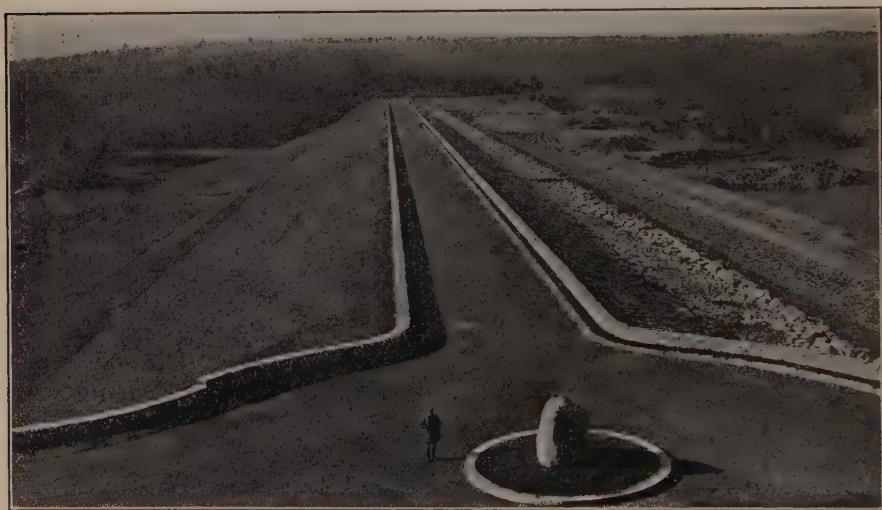


FIG. 9. — QUABBIN DIKE — ROADWAY ACROSS TOP OF DIKE, LOOKING WESTERLY FROM THE EAST END

cated the necessity of a cut-off to rock as at the Main Dam. Although the materials in these foundations were suitable for safely supporting the weight of the structure, their highly permeable character required that a tight cut-off wall be provided, both for security and to save the water which would be otherwise lost through them. The caisson cut-off was built under a contract with the West Construction Company at a cost of \$939,695.78. In addition, the exploratory caisson, which is part of the finished cut-off, together with pumping, cost about \$100,000. An open trench from 20 to 40 feet deep was excavated across the valley, and the caissons were sunk to ledge from the bottom of this trench.

Thirty-one caissons, 45 feet long and 9 feet thick, except for the exploratory caisson, were sunk and the spaces between them excavated and filled with concrete. The bedrock below the caissons was grouted and the caissons were sealed to it. The caissons projected, in general, about 25 feet above the bottom of the open-cut trench, and the space around them was sealed with impervious refill formed by placing topsoil in 6-inch layers consolidated with heavy sheep's foot rollers. As at the Main Dam, the lowering of ground water aided the sinking of this core wall materially, and thereby greatly reduced the cost of it. For effect of pumping upon air pressures used during sinking operations, see the table accompanying the description of work at Quabbin Dam.

The contract for the caissons took the greater part of the construction seasons of 1933 and 1934.

The contract for the embankment was let to the Arthur A. Johnson Corporation on November 26, 1934, at a contract price of \$1,446,655, and was completed on December 31, 1937. The work under this contract consisted of completing the consolidated foundation embankment in preparation for the hydraulic sluicing, placing about 2,000,000 cubic yards of fill by full-hydraulic sluicing methods, construction of the top of the Dam over the hydraulic work by rolled fill methods, and the placing of soil dressing on the downstream slopes and riprap on the upstream slopes. Materials for the hydraulic embankment were obtained from a borrow pit on the westerly side of the Beaver Brook Valley at an average distance of about one half mile from the westerly end of the Dike. These materials were excavated with electric and gasoline shovels and transported to the site by wagon units equipped with side boards to carry 8 cubic yards. Materials were carried to a special mechanical mixing plant or hog box which consisted of an unloading deck, upon which the trucks ran, built in an elevated position over receiving hoppers. These hoppers emptied into feeding machines which regulated the amount of material to be sluiced on to the dam. From these feeders, the material, after passing through a rotary grizzly which removed the sizes over 7 inches and discharged them into a boulder chute, passed into a sluiceway beneath. The water was fed through this sluiceway at a high velocity, carrying the materials into the sluicing line. This line passed through a dredge pump which provided any deficiency in pressure needed to force the materials on to the Dike. The discharge lines were 20-inch welded steel pipes. Work was carried on with three 8-hour shifts per day, six days per week, and the rates of placing the hydraulic embankments were generally from 48,000 to 50,000 cubic yards a week. The concentration of solids in the sluicing

mixture was generally from 13 to 15 per cent. The method of placing the material in the hydraulic embankment after it left the dredge pump into the sluicing lines was substantially the same as described in greater detail in the description of the Main Dam. After the hydraulic fill had been carried to within about 30 feet of the top of the Dam, the embankment was completed by rolled-fill methods, the core being built of impervious glacial till placed in layers and consolidated with a sheep's foot roller, and the shoulders of coarser glacial materials generally consolidated by dumping into water.

CONCLUSION

Quabbin Reservoir, with its long-time storage and great depth, will furnish water of excellent quality. Even in the early years before the reservoir is seasoned, no difficulty need be anticipated which cannot be easily corrected, and the additional period of storage in Wachusett Reservoir will insure the continued good quality of water which enters the Wachusett Aqueduct at Wachusett Dam.

An approximate measure of the pollution hazard on the various watersheds used for Metropolitan water supply is indicated by the population per square mile on the various drainage areas, which is as follows:

Cochituate Watershed, first used in 1848	1,225
North Sudbury, developed from about 1875 to 1908	550
South Sudbury, developed about 1875 to 1895	230
Wachusett, developed 1895 to 1908	74
Ware, first used in 1931	34
Swift, estimated upon completion of the work in 1940	8

It was recommended in a special report of December 1, 1937, of the "Metropolitan District Water Supply Commission and the Department of Public Health Relative to Improvements in Distribution and to Adequate Prevention of Pollution of Sources of Water Supply of the Metropolitan Water District," that a new pressure aqueduct be built from the terminal chamber of the masonry portion of Wachusett Aqueduct to the upper end of the Weston Aqueduct at the Sudbury Dam; that the Weston Aqueduct be developed to its ultimate capacity by building an additional pipe line at each of the two siphons at Sudbury River and Happy Hollow; that a section of pressure aqueduct be built from near the end of Weston Aqueduct to Chestnut Hill Reservoir; and that a by-pass pipe line be built around the southeast side of Spot

Pond, connecting the pipe line from Weston Aqueduct to the Spot Pond pumping station.

Upon the completion of such works, the further use of water from the Sudbury and Cochituate drainage areas and the use of the Sudbury and Cochituate aqueducts would be discontinued, and only water from the Wachusett Reservoir, supplemented by the Ware and Swift River sources to the west, would be used to supply the District; also, Chestnut Hill Reservoir and Spot Pond would be used only in case of emergencies.

With the elimination of the Sudbury and Cochituate sources, the Swift-Ware-Wachusett supplies will yield safely 293,000,000 gallons per day, which is estimated to be adequate to serve the Metropolitan Water District and all probable additions thereto until about the year 1975. When additional water is required, it may be obtained at relatively small expense by diverting water from other streams into Quabbin Reservoir for storage in a manner not to seriously injure riparian owners thereon.

The said report also covered the results of an investigation of sources of pollution to the water supply and recommended their elimination.

Chapter 460, Acts of 1938, approved June 28, 1938, provides for carrying out the recommendations of this Joint Board. The works to be constructed under this act, as above outlined, will deliver water at Chestnut Hill Reservoir at about Elevation 195, or over 60 feet higher than this reservoir. The pumps at Chestnut Hill Reservoir will take their supply directly from the pressure tunnel and the pumps at Spot Pond directly from the new pipe lines, thus eliminating these reservoirs from regular use and reducing the lift on the pumps.

The works to be built under this act form a part of a comprehensive tunnel distribution loop through the Metropolitan District, which, when built, possibly fifteen years hence, will deliver water in the vicinity of Chestnut Hill Reservoir about 135 feet higher than this reservoir, and in the vicinity of Spot Pond Reservoir, fully 100 feet higher than that reservoir. Pumping will no longer be required except to certain high points in the District, such as Arlington Heights, Bellevue, Lexington, Bear Hill, etc., and only about 5 per cent of the entire supply will need to be pumped.

COST OF WORK

There had been expended to June 30, 1938, on the Ware-Swift project, including a variety of other work which has been assigned from time to time to the commission, the expense of which is included, a sum of \$42,656,829.86. The amount provided in chapter 375, Acts of 1926, and chapter 321, Acts of 1927, was \$65,000,000, to which may be added the revenue collected from the various sources, such as rents, sales of timber, etc., about \$910,036.34.

It is estimated that there will be sufficient surplus from the amount provided for the Ware-Swift water supply to build all the work provided for in chapter 460, Acts of 1938, above outlined.

Descriptions of many features of the work have been omitted, such as details of design, methods of construction, methods of real estate and topographic surveys, etc. The following bibliography, which is believed to be reasonably complete, may be consulted for much of the information omitted.

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TESTS FOR THE PERMEABILITY OF SOILS

BY CHARLES W. ROBINSON, MEMBER *

INTRODUCTION

THIS paper was written with the idea in mind of giving an account of the operations necessary for getting permeability values. The published articles on the subject do not go into details. A description is given here of actual tests and the precautions taken to insure accurate results. It was written, not for experts, but for those not entirely familiar with the laboratory work necessary to get the coefficient of permeability, or with its use after it is found. A description of the use of piezometers for measuring the influence of various factors on the flow and on the structure of the sample is given. As far as we know, the employment of piezometers for getting conditions within the sample is original.

The rate with which a soil will permit water to flow through it is of prime importance in the design of practically all structures in contact with earth. It is of especial importance in the design of earth-dams, foundations on more or less plastic soils, retaining walls and cofferdams.

The importance of getting accurate values led us to carry on a series of tests in our laboratory. Investigation was made of several types of permeameters, methods of filling, and ways of handling the sample. A special permeameter was made with side tubes, or piezometers, to measure the pressure heads in the sample. It is believed that an account of our experiments with a few general notes on the subject will be of some interest and value to those who have to make decisions based partly or wholly on laboratory data.

The coefficient of permeability, k , is the velocity of flow through a cross section of the soil, including both voids and solids, under a hydraulic head equal to unity. It is affected by the temperature and the percentage of voids in the cross section. For the reason that the "Centimeter-gram-second" system is used in soil research as in other scientific lines, k is usually given in centimeters per second.

The hydraulic gradient used in soil physics is somewhat different from that used in hydraulics. In soil research the hydraulic gradient is

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the total head acting on the sample divided by the length of the sample, taken parallel to the flow. The pressure head and corresponding gradient may be found at any point in the sample by means of piezometers entering the side of the permeameter.

To get a value indicating the degree of compaction of a soil corresponding to values of k , its void-ratio is calculated. This is the volume of voids in the sample divided by the volume of the solid portion. This

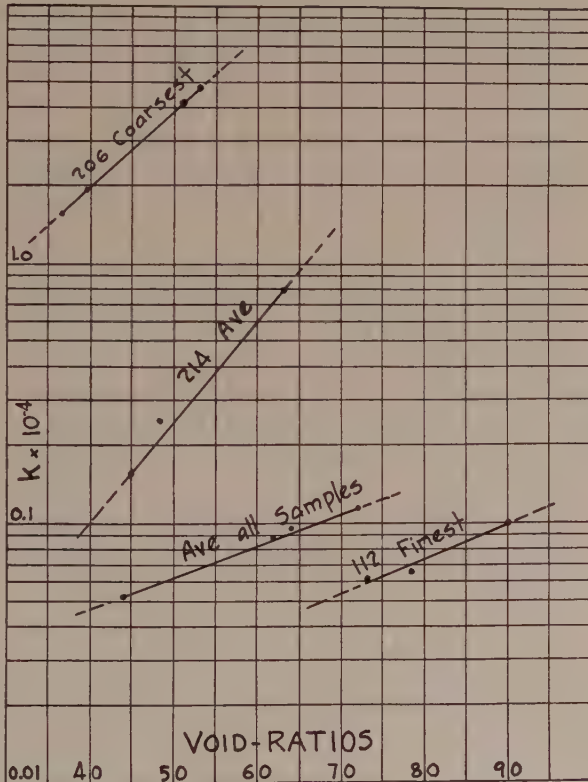


FIG. 1

value, e , is usually expressed as the ratio of the volume of voids to the volume of the solid. The volume of the solid is found by dividing the dry weight of the soil by its true specific gravity. The volume of the solid portion subtracted from the total volume gives the volume of the voids. The ratio of the volume of the voids to the total volume is the porosity, n . This value should not be confused with the void-ratio,

and when making reports it should be definitely stated which value is being used. The void-ratio is based on the solid phase, which is a constant, while the porosity is based on the total volume, which is constantly changing.

Permeability values plotted against void-ratios on semi-logarithmic paper will give a series of points through which a straight line may be drawn. This makes interpolation easy. The k values are plotted on the log scale, and the corresponding e values on the uniform scale. Having three or more points connected by a straight line, the corresponding values of k for other degrees of compaction as denoted by e may be found by interpolation, extending the line if necessary. (See Fig. 1.)

The viscosity of water increases as its temperature drops, and, other things being equal, the rate of flow decreases. Accurate readings of the temperature are therefore very necessary when getting values of k .

True specific gravity also must be determined carefully, as on it the value of e depends. The mineral constituents of a soil may vary considerably, even in the same general locality. Soils in which quartz predominates usually run around 2.65, while those containing more or less ferro-magnesian material will probably run to 2.80 or over. Some clay soils are light because of their kaolin content.

The sample should be air dry and all lumps reduced by rolling with a rubber roller or rubbing lightly through a sieve. Oven-dry material seems to have lost some of its natural character, and does not act just the same as soil in its natural state. A portion may be oven-dried to get the percentage of moisture in the air-dry material, or the sample may be oven-dried after the test and weighed. The latter is often the better procedure.

After the sample is air-dried and broken down it should be well mixed to a uniform composition, and, in putting it into the permeameter, much care must be used to prevent its forming segregations of coarse or fine particles. It should be placed in layers and tamped to the desired degree of compaction. Each layer should be indented a little with the rod before putting in the next layer. This is to prevent the formation of lines of easy flow or channels to the sides of the permeameter or leading through some part of the cross section of the sample. A long-handled spoon is useful for placing the soil in the permeameter without dropping it any appreciable distance, and for keeping the particles well distributed.

If the permeameter is to be filled with dry soil, it is advisable to stand it in a dish containing a little water so that capillarity will act and the water rise through the soil and drive out the air as the perme-

ameter is filled. It may be necessary to wait now and then for the water to catch up. If this is not done with fine material, the sample will be pulled together radially when the water is allowed to enter, and an annular space will appear between the inner surface of the permeameter and the outer surface of the sample. This will be hard to get rid of without forming channels. When this happens it is best to empty the permeameter and refill, using the now saturated soil.

The sample may be placed either dry or saturated as convenient. If proper precautions are taken the results are satisfactory either way. There is less likelihood of segregation when handling saturated material. To saturate the sample it is first well mixed in an air-dried state and placed in a sieve fine enough to retain it without loss. The sieve containing the soil is set in a dish in which there is water just deep enough to reach the bottom of the soil, and kept at about that level. After the water has risen to the top of the soil it is left in the dish some time to soak the air out. It is then removed and set aside to drain until no free water runs out when it is handled with a spoon. It should be just wet enough to cling together well.

With large permeameters requiring large amounts of soil, it is convenient to fill with dry material for the first run, tamping it rather lightly to get a large void-ratio. After this run, the soil, now saturated, is dumped out and put back again, with a little harder tamping than for the first run. This procedure is repeated until a sufficient number of values of void-ratio and k are obtained. These are plotted on semi-log paper, as previously explained.

Sets of values may also be obtained by using a number of small permeameters. The sample is divided and a portion placed in each. Each portion is tamped a little harder than the previous one so that a series of decreasing values of e and k is obtained. After the test, each portion is put in a numbered dish, dried in the oven and weighed. Care is taken to wash all of the soil out of the permeameter into the dish in each case.

Water used must be as free from air as possible. Distilled water, or water that has been boiled, may be used, but water that has stood for a long time with occasional stirring will be practically free from air bubbles. Large storage vessels of some kind are therefore useful in a soils-mechanics laboratory. The water should be also fairly pure, as mineral salts, organic acids or suspended matter may act on the soil and influence the flow through it.

A small amount of humus in a soil to be used as core material may be negligible and some soils may need a little. On the other hand,

any considerable amount may prevent proper drainage and reduce the stability of the core as well. As it will not be used in the construction of the dam to any extent, it is well to remove it from the samples before any of the tests are made. It has a marked effect on both the mechanical analysis and the permeability tests. To remove the humus the sample may be treated with N/5 hydrochloric acid to dissolve the humates. Water containing strong ammonia in the proportion of 2 c. c. of ammonia per liter of water may then be run through to remove the acid.

Modern methods of soil analysis enable one to make mechanical analyses quickly and easily. Plotting the results of the analyses on

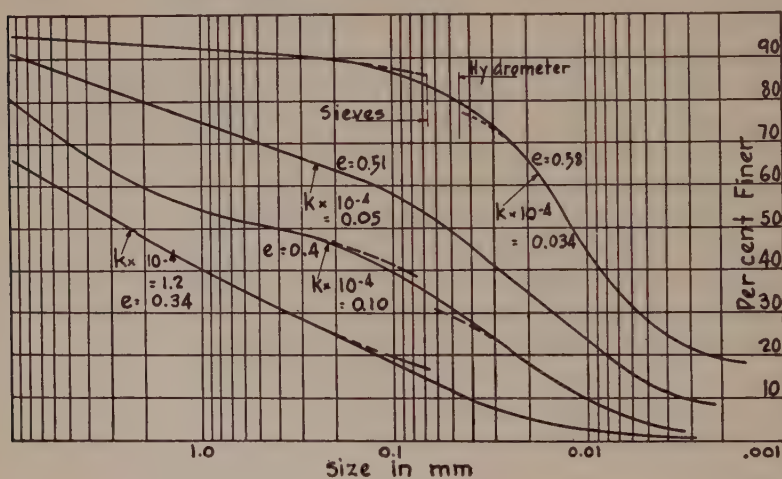


FIG. 2

semi-logarithmic cross-section paper, there are developed gradation curves which give some idea of the permeability of the soils represented. Two values which may be obtained from a gradation curve have been in use for some time in formulas for getting permeability without making direct tests; one is the "effective size" and the other is the "coefficient of uniformity." These values were first suggested by Allen Hazen. Effective size is the size, or diameter, at the 10 per cent line on the curve. The uniformity coefficient is the diameter at the 60 per cent line divided by the effective size. Effective size without doubt governs the flow of water through the soil to a great extent. We have found, however, that even with soils having practically the same effective size and uniformity coefficient, the permeability will

vary considerably. The flow is dependent not only on the fines, but also on the amount and gradation of the coarser material.

Other factors enter into the problem. Among the most important are composition, shape and arrangement *in situ*, or natural structure. These factors, of course, cannot be obtained from the gradation curve. From that curve we may get some knowledge of the natural density and the sizes of the voids, but beyond that it does not help us. The compound microscope, to a limited extent, will reveal the shape and mineral composition of the particles, and from the minerals we may get the chemical character of the soil. The liquid limit tests of Atterberg yield useful data concerning the character of soils and their reactions with water. Data from these various sources serve to divide soils into groups and so reduce the number of permeability tests required. A certain number will still be necessary, however, for a satisfactory knowledge of k . A more extensive study of the relationships existing between gradation curves, liquid limit tests, and permeability would doubtless be of considerable value. (See Fig. 2.)

Some years ago we used the settling out or sedimentation process for the analysis of fine soils, checking sizes as far as possible with the compound microscope. We now use the hydrometer method as perfected by Dr. Arthur Casagrande, with an immense saving of time and equal accuracy. We believe that air-dried materials give better results than oven-dried. The moisture content of the air-dried soil may be determined and subtracted from the weight of the sample before the analysis, or the sample may be weighed afterward. Too much beating during the dispersion breaks up the particles and increases the amount of fines and should be avoided. No more of the deflocculating agent should be used than is found necessary by experiments on the soil under observation. It is a good plan to let the sample soak awhile after dispersion. We usually beat up the fine portion of the sample with some of the deflocculating agent and let it set over night. Just previous to the hydrometer readings we disperse it again by beating and shaking. Several analyses may be run at the same time by one operator.

Borrow-pit samples as received in the laboratory for analysis have all of the natural constituents of the soil in place, except, perhaps, the water content, and that varies with the weather. The fact that they are disturbed so far as structure goes, makes no difference in the results, as the material in the dam will have little, if any, structural relation to the material in its natural state. The soil under the dam will be much compressed and altered and will differ considerably in structure

from what it was previous to the placing of the dam; therefore disturbed samples are satisfactory for most purposes, in preliminary work, at least.

Our experimental permeameter is an iron cylinder 30 cm. high by 13 in diameter, with a screen about 5 cm. up from the bottom. The screen supports the sample which extends up to about 6 cm. below the top of the cylinder. There is a side opening into the space under the

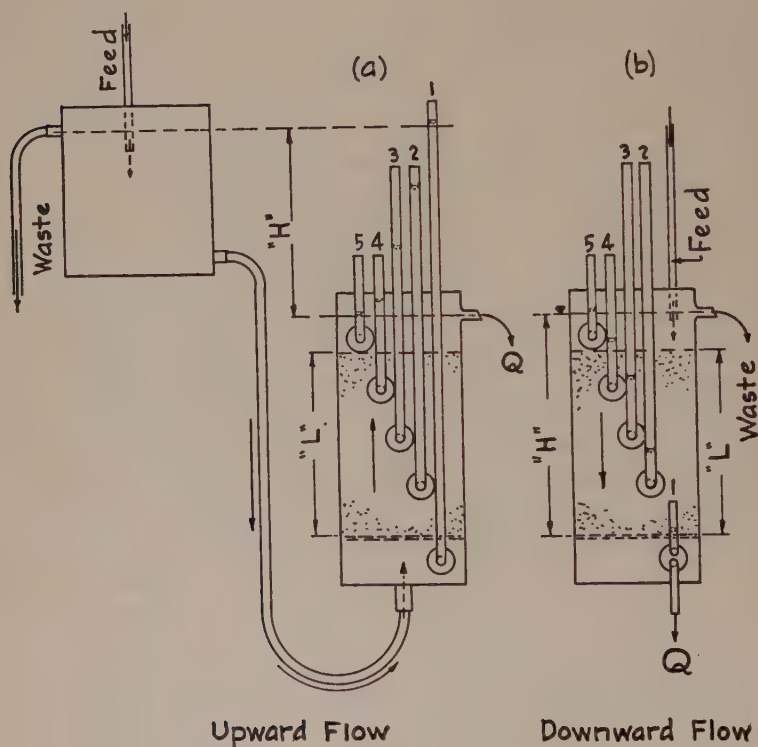


FIG. 3. — CONSTANT HEAD PERMEAMETER

sample and one above the sample in the free water. There are three openings into the soil compartment, making five vertical openings or orifices in the side of the permeameter. There is one horizontal orifice in the bottom for the entrance of water in upward flow tests. (See Fig. 3.) An exit 1.5 cm. below the top acts as spillway, and water from this exit is collected and measured for Q during upward flow tests.

During downward flow, this exit keeps the head constant, and the flow is collected at the bottom and measured. At the beginning of the tests the water was collected as it came through the tube connected to the bottom orifice. It was found, however, that this bottom tube acted as a draft tube and reduced the pressure heads abnormally and affected the flow quite noticeably. Thereafter use was made of the side orifice under the sample as an outlet, and a uniformly consistent discharge was obtained. Q from downward flow was taken in a number of ways which will be described farther on.

For the purpose of getting pressure heads, a glass tube, or piezometer, was inserted in each of the side orifices. The tube from the space under the sample registered full head during upward flow, minus a small entrance loss. The tube from over the sample at the top would, of course, register zero, or tail-water during upward flow. The tubes between registered pressure heads at three points in the sample, and showed that the losses between the ends of the sample did not vary as a straight line. A part of the deviation was probably due to losses at the entrances to the piezometers.

A study of Table 1 will show that the deviation from a straight line, or uniform loss of head between the ends of the sample, was small. This was probably due, in part, to exit loss to the piezometers. When the deviation was large it indicated that changes had taken place in the sample. In this case the sample was taken out and replaced after being thoroughly remixed. Changes were usually due to a washing out of the fines or quicksand effects of high heads. The influence of other factors could also be measured, such as, for instance, the draft-tube action of an exit tube attached to the bottom exit and channels formed in the sample by segregations of the coarser constituents.

TABLE 1. — SUMMARY OF RESULTS OBTAINED WITH CONSTANT HEAD PERMEAMETER

(See Fig. 3)

Flow	H cm.	h_1	h_2	h_3	h_4	h_5	e	k	Soil
Upward . . .	19.3	17.3	12.0	7.0	3.3	0.0	0.53	0.0400	(a)
Upward . . .	19.5	19.5	12.0	6.5	1.0	0.0	0.44	0.0260	(a)
Upward . . .	9.3	8.0	6.0	4.0	2.0	0.0	0.70	0.0800	(a)
Upward . . .	9.3	9.3	5.7	2.7	0.6	0.0	0.48	0.0220	(a)
Upward . . .	9.3	9.3	5.7	2.7	0.5	0.0	0.40	0.0128	(a)
Downward . .	24.7	0.0	10.2	16.0	21.1	24.7	0.68	0.0475	(a)
Downward . .	24.7	0.0	6.2	13.3	20.3	24.7	0.56	0.0316	(a)
Downward . .	24.7	0.0	8.1	16.1	23.7	24.7	0.42	0.0185	(a)
Downward . .	24.5	0.0	9.8	16.3	22.3	24.5	0.43	0.0297	(a)
Downward . .	24.5	0.0	11.0	16.8	21.9	24.5	0.43	0.0210	(a)
Downward . .	25.5	0.0	7.5	13.3	20.2	25.5	0.79	0.0530	(b)
Downward . .	25.5	0.0	8.0	13.7	19.7	25.5	0.53	0.0350	(b)
Downward . .	25.5	0.0	10.7	19.4	24.7	25.5	0.32	0.0045	(b)
Downward . .	25.5	0.0	10.6	19.4	23.1	25.5	0.54	0.0320	(b)

Before the upward flow starts, that is, before the head is allowed to act on the sample, the water in all the side tubes stands at the level of the tail-water over the sample. After flow is started by allowing the water to enter through the bottom orifice, the water levels drop to points corresponding to pressure heads at points within the sample.

During downward flow the side tube under the sample is omitted and that orifice is used as an exit. The tube above the sample registers headwater and the losses may be measured directly between headwater and pressure heads within the sample, as indicated by the water levels in the side tubes, or piezometers.

The first tests were made with upward flow. The head reservoir has an overflow orifice to maintain a constant level. (See Fig. 3.) It is fed from an overhead tank, just fast enough to allow a slight waste and insure constant pressure. The headwater in the head reservoir was 19.5 cm. above the tail-water in the top of the permeameter; that is, the total head, H , was 19.5 cm. The pressure from this head was found to be too great. It washed out fines and formed channels, and the top of the sample took on the consistency of quicksand. These effects were not so noticeable in hard tamped soil as in samples put in with light tamping for the purpose of getting large void-ratio values.

An attempt was made to put a 10-pound weight on the sample to prevent quicksand effects. The weight had a tendency to settle the soil while the water was running, and the result of this was a decrease in the flow under the same head and temperature due to the gradual compacting of the sample. This action ceased with the removal of the weight. The headwater was then lowered to 9.5 cm. above tail-water. This reduced the hydraulic gradient from about 1.0 under the 19.5 cm. H to about 0.5 with the 9.5 cm. H . The results under the reduced gradient were consistent and constant for each void-ratio, except, of course, for variations due to temperature. All values were corrected to 20 degrees Centigrade. (See Table 1.)

At the beginning of the "downward-flow" experiments, the bottom of the sample was left free, that is, the cylinder was not immersed and there was no fixed tail-water. Zero pressure would still be at the bottom of the sample. Before flow started, the meniscus in each tube stood at the level of the headwater. After flow started, it dropped to a level corresponding to the level of the pressure head at the point of exit. The loss was then measured from the pressure head to head-water. The results, with the bottom of the sample free, were a bit inconsistent, due perhaps to draft effect. The bottom of the sample was now immersed in water definitely fixing the level of the tail-water at about that of the bottom of the sample. With this arrangement the results were consistent and satisfactory. Next, the cylinder was taken out of the water and a two-hole stopper was placed in the orifice under the sample in the side of the cylinder. In one hole there was inserted a glass tube bent up to act as a standpipe; in the other hole there was placed a short straight tube for outflow. After flow started, the water in the standpipe showed that tail-water, or zero pressure, was at the bottom of the sample. With the two-hole stopper arrangement practically the same results were obtained as with the permeameter standing in water. The results of the "down-flow" experiments are given in Table 1.

Results obtained from previous experiments with types of permeameters, varying from complicated to simple, and from later experiments with the large permeameter as just described, lead us to believe that downward flow is to be preferred to upward flow for permeability tests, except perhaps when they are made in combination with compression tests. Also the variable head permeameter is preferred for routine work.

The first experiments were made with a heavy brass permeameter, with a well-fitted piston and porous stones in the piston over the sample

and in the bottom under the sample to allow the escape of water. Any specified pressure may be put on the sample in this apparatus, and k , e and moisture content obtained at that pressure. It has several additional features for use when getting k and e only, and is excellent for experimental purposes but much too complicated for ordinary work.

We now use a set of six pyrex tubes fitted with standpipes and screens. One, 30 cm. high and 9.5 cm. in diameter, we use for soil containing stones up to about 1 inch in size. Five are 15 cm. by 4.5 cm. each, and are used for soils ranging from gravel to fine clay. With this set of five there is obtained a series of values for k and e from each sample from one set-up. These values are plotted on semi-logarithmic paper. From the line drawn, which may be produced in either direction, may be found a k value for any void-ratio assumed, computed or obtained in the field from undisturbed samples. When using the large glass permeameter for coarse, rather freely permeable soil, we time the flow between heads in the upper part of the tube itself, and compute k by means of the same formula for variable heads that we use with the smaller tubes and standpipes. Table 2 gives values obtained with this set of glass tube permeameters. It may be noted that a series of values was obtained on the same soil, listed as (b), with Permeameters A, B, C and D, in Table 2. The analyses of these soils are given in Table 3.

Observations on a series of long-time runs seem to indicate that when the flow through the sample is allowed to continue for some time, there is a gradual movement of fines from the line of maximum pressure to the line of zero pressure. These fines build up at some point in the sample and decrease the flow for a while until they are pushed along and finally emerge from the sample. The progressive building up and breaking down of the plugs, or filter skins, naturally results in a fluctuation in the flow, or discharge.

TABLE 2. — SUMMARY OF RESULTS OBTAINED WITH VARIABLE HEAD PERMEAMETERS
(See Fig. 4)

PERMEAMETER	Void-ratio e	k in Cm. per Sec.	Soil
A	0.54	0.02000	B1
A	0.525	0.01848	(b)
B	0.460	0.00740	H1
B	0.500	0.01450	(b)
C	0.710	0.00760	B2
C	0.510	0.01540	(b)
D	0.690	0.00500	H2
D	0.450	0.01010	(b)
E	0.730	0.000315	VBH
F	0.750	0.014160	H1
F	0.480	0.007400	(b)

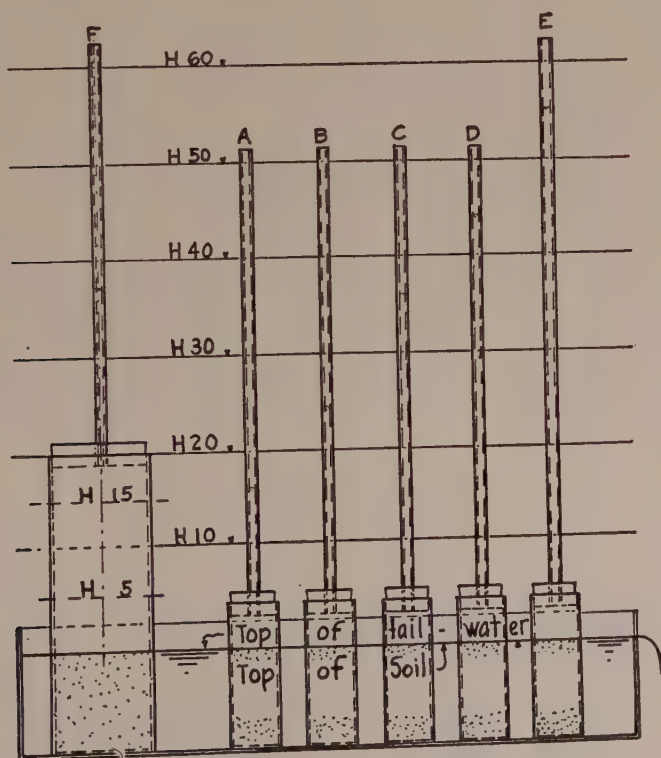


FIG. 4. — VARIABLE HEAD PERMEAMETERS (RESERVOIR NOT SHOWN)

TABLE 3.—RESULTS OF SIEVE ANALYSES OF SOILS OF TABLE 2

SIEVE NO.	Size (mm.)	PER CENT PASSING EACH SIEVE							
		a	b	c	B1	H1	B2	H2	VBH
¾	18.55	—	—	98	—	—	—	—	—
⅝	9.42	—	100	79	—	—	—	—	—
4	4.76	100	94	71	100	100	—	—	—
8	2.38	91	81	66	90	85	—	—	—
16	1.19	74	66	52	75	67	—	—	—
30	0.59	39	36	29	43	40	100	100	100
50	0.297	12	16	13	17	22	40	56	74
100	0.149	3	5	5	3	6	6	16	58
200	0.074	1	2	1	2	3	4	6	29
300	0.046	0.5	1	0.5	1	2	2	4	23

A similar action doubtless takes place in an earth dam, with the exception that fines are coming in at one end of the path of flow while they are being pushed out at the other. In a well-designed dam, the chances are that equilibrium will finally be established, and the dam will be tighter than at the beginning of flow.

A series of long-time runs with a permeameter having side tubes for getting internal pressures and fed with fine material to take the place of that being washed out, when completed, may give us some useful data in regard to the gradation required to reduce or stop the loss of fine material.

Formulas in about the form given by Dr. Glennon Gilboy are used for computing the coefficient of permeability from data obtained from laboratory tests.

For constant head permeameters there is used —

$$Q = k i A T$$

in which —

Q = the total discharge.

k = the coefficient of permeability in centimeters per second for a hydraulic gradient equal to one.

i = the hydraulic gradient, = H divided by L .

A = the area of the sample at right angles to the flow.

T = the time in seconds taken to produce Q .

The manner of getting Q is explained in connection with the experiments made with the large permeameter and piezometers. All values should be corrected for temperature to some standard, say 20° C., so that they may be comparable.

For variable head permeameters the formula is —

$$k = \frac{a L}{A T} \log_e \frac{H1}{H2}$$

in which —

k = the coefficient of permeability in centimeters per second.

a = the area of the standpipe in square centimeters.

A = the area of the sample in square centimeters.

L = the length of the sample in centimeters.

T = the time in seconds for the water level in the standpipe to drop from $H1$ to $H2$.

$H1$ = the head taken as the upper limit, in centimeters.

$H2$ = the head taken as the lower limit, in centimeters.

When values of k are very small, as the lower values in Table 4, they may be multiplied, for convenience, by some power of 10. Take from Table 4 the value of k , 0.0000097 for instance, and multiply by 10^4 , or 10,000. The product is 0.097, an easier number to handle. The value is stated as $k \times 10^{-4} = 0.097$ cm. per sec. This value may be plotted against values of void-ratio on semi-logarithmic paper just as it is, and may be quickly and easily converted into any unit of measure desired. Given a value $k \times 10^{-4} = 0.8$ cm. per sec. To get meters per day, place four ciphers between the decimal point and the first figure, multiply by the number of seconds in a day, and divide by the number of centimeters in a meter.

$$\frac{.00008 \times 60 \times 60 \times 24}{100} = .069 \text{ meter per day}$$

To get feet per day, multiply by the conversion factor 3.28. The product is 0.23 foot per day. If the area through which the loss occurs is taken in square feet, a speed of 0.23 foot per day will produce a loss of 0.23 cubic foot per square foot per day. To get gallons per square foot per day, multiply 0.23 by 7.5, the number of gallons of water in a cubic foot.

Table 4, which follows, gives some typical examples taken from a large number of runs on test pit samples from a recent earth-dam project. Four permeameters were used to get a set of values for each sample tested. The first one was filled lightly, and each successive permeameter was packed a little harder than the one preceding it. Four void-ratios were thus found with corresponding coefficients of permeability. The fifth permeameter was a heavy brass cylinder used principally for compression tests. With this cylinder any desired load may be applied to the sample and the percentage of water determined at the maximum consolidation under the specified loading.

TABLE 4. — SOME TYPICAL EXAMPLES OF k VALUES

SERIES	Sample	e	k in Cm. per Sec.	Notes
200	206	0.53	0.000485	Coarsest of series
200	206	0.51	0.000410	Coarsest of series
200	206	0.40	0.000195	Coarsest of series
200	206	0.37	0.000158	Coarsest of series
200	206	0.34	0.000129	Same under 30 lbs. per sq. in.
200	Ave.	0.63	0.000080	Average of series
200	Ave.	0.58	0.000049	Average of series
200	Ave.	0.49	0.000025	Average of series
200	Ave.	0.45	0.000016	Average of series
200	Ave.	0.40	0.000010	Same under 30 lbs. per sq. in.
Composite	—	0.72	0.0000133	200+100 combined average
Composite	—	0.64	0.0000097	200+100 combined average
Composite	—	0.62	0.0000089	200+100 combined average
Composite	—	0.58	0.0000058	200+100 combined average
Composite	—	0.51	0.0000046	Same under 30 lbs. per sq. in.
100	112	0.90	0.0000100	Finest of series
100	112	0.79	0.0000065	Finest of series
100	112	0.73	0.0000060	Finest of series
100	112	0.58	0.0000034	Same under 30 lbs. per sq. in.

OF GENERAL INTEREST

JOINT EXCURSION TO QUABBIN RESERVOIR — BOSTON SOCIETY OF CIVIL ENGINEERS, AMERICAN SOCIETY OF CIVIL ENGINEERS, NORTHEASTERN SECTION AND SANITARY SECTION, BOSTON SOCIETY OF CIVIL EN- GINEERS

An excursion by members of the Boston Society of Civil Engineers, the American Society of Civil Engineers, Northeastern Section and the Sanitary Section of the Boston Society of Civil Engineers was made on Saturday, June 11, 1938, to the Quabbin Reservoir and other features of the Ware-Swift additional water supply for the Metropolitan District, under the direction of the Metropolitan District Water Supply Commission. More than two hundred members and guests, including ladies, made the trip in buses and private automobiles.

In spite of numerous showers, the party had an opportunity to inspect the very interesting features of the work already completed as well as that under construction. Although the plans had provided for lunch at Shaft No. 12, on the shore of the present Quabbin Lake, the rain necessitated other arrangements, and the officials of the Commission directed the party to the village of Dana, where the town hall, church and other facilities were made available for serving box lunches.

Ware River Intake Works

The Ware River Intake Works, at the top of Shaft No. 8 of Quabbin Aqueduct, was selected as the assembly point for all making the excursion. At this point President Karl R. Kennison, Assistant Chief Engineer of the Commission, outlined the water supply system and the purpose of this intake, and described its construction and operation.

He stated that the route from Boston to the Ware River Intake Works passed in the vicinity of many of the reservoirs of the Metropolitan Water District. Lake Cochituate first supplied Boston through Cochituate Aqueduct in 1848, but is now seldom used. The reservoirs on Sudbury River, built 1872-1898, consist of Nos. 1, 2, 4, 6 and 8, the latter three known as Ashland, Hopkinton and Whitehall reservoirs, respectively, on the South Branch and now seldom used, and Nos. 3 and 5, the latter known as Sudbury Reservoir, on the North Branch. These, and Sudbury Aqueduct, were all built by the city of Bos-

ton, before the formation of the Metropolitan Water District, except Sudbury Reservoir, completed in 1898 by the Metropolitan Water Board, which took over the system in that year.

This Board constructed Wachusett Reservoir farther west, on the South Branch of Nashua River, which was first filled in 1908. Water from this reservoir flows through Wachusett Aqueduct into Sudbury Reservoir and thence follows the aqueducts and pipe lines to distributing reservoirs at Chestnut Hill and Spot Pond. In 1904, to supplement Sudbury Aqueduct, Weston Aqueduct was completed from Sudbury Reservoir to connect with Chestnut Hill. A direct connection to Spot Pond was made in 1926. The water from Ashland, Whitehall and Hopkinton flowed through Nos. 2 and 1 until 1927, when Whitehall and Hopkinton were diverted into Sudbury Reservoir, and Ashland into Sudbury Aqueduct.

Extension of the sources to Ware and Swift rivers was commenced by the Metropolitan District Water Supply Commission late in 1926. The water will flow by gravity from Quabbin Reservoir to Wachusett Reservoir through Quabbin Aqueduct, a 24.6 mile tunnel through solid rock, twice as long as the Simplon Tunnel under the Alps. The Ware River Intake Works, completed in 1931, is at the top of Shaft No. 8 of Quabbin Aqueduct. The tunnel is 271 feet underneath the building. Flood flows from the Ware are dropped into the tunnel and diverted about 13 miles easterly to Wachusett Reservoir, to an extent limited by the present storage capacity of that reservoir. Flow in the tunnel will be controlled by valves at the Wachusett end. When these valves are closed Ware River water will flow westerly to Quabbin Reservoir. Also later, if desired, the Quinapoxet River, the main feeder of Wachusett Reservoir, may be diverted into the tunnel

at Shaft No. 2 and thence to Quabbin Reservoir. The top of the shaft is sealed with the concrete floor which forms the bottom of the valve pit. The pit is dry during the summer months, since the flow in the river is ordinarily less than the diversion limit of 85,000-000 gallons daily. The diversion into the pit is made through nine siphon spillways, which are primed by sensitive float valves in a pit in the basement floor, connected to the vacuum that is created in the Venturi meter which measures the 85 m. g. d. discharge into the river.

The four main butterfly valves of the Dow disc-arm type are operated by oil pressure, the oil being distributed to the opening or closing pipes through four sleeve valves in the rear of the operating panel in the corner of the main floor. Blue prints on the walls show the details of operation, which is entirely automatic. The cams which operate the oil sleeve valves are actuated not only from a float well in the pit, but also by cables which are connected to the cross heads of the butterfly valves and which extend up through the swirling water to a compensating mechanism behind the operating panel, which prevents "hunting" and insures steady operation. The air compressor in the basement not only maintains the necessary pressure in the oil and water tanks, but is also connected to an air lift embedded in the lining of the shaft from which the water supply is obtained.

From this point the party went to Dana for lunch.

East Branch Baffle

The next point of interest was the East Branch Baffle, an earth dike protected by heavy riprap on both sides, which will deflect to the north the water discharged into the reservoir from Shaft No. 11-A, so that the water must travel

11 miles through the reservoir and have the benefit of long-time storage before being drawn at Shaft No. 12 into the aqueduct to Wachusett Reservoir. The gap at the north end of the Baffle will be filled as soon as traffic on old Route 21 can be discontinued. The river now flows through a concrete conduit which will be shut off by stop logs placed at the steel-covered structure alongside the temporary highway on the upstream berm. The reservoir to the north has been deepened by a cut through the divide so that 50 feet of storage will be drawn to the north. The closure of the conduit through the Baffle will be permanent, except that the logs can be lifted to draw on the small storage, if ever needed, below this 50-foot depth.

Intake to Quabbin Aqueduct — Shaft No. 12

The main intake from Quabbin Reservoir to Quabbin tunnel is at the so-called Shaft No. 12, near the small present Quabbin Lake. The intake works are situated at the top of this shaft. At this building, Mr. Frank E. Winsor, Chief Engineer of the Commission, described the equipment and functions at this main control point in the system.

The intake structure is built at the top of Shaft No. 12, 125 feet above the tunnel. The finished grading of the level area in front of the building will be 6 feet above the reservoir flow line. The low-level intake can be seen at the foot of the hill. It consists of a coarse concrete rack above the entrance to a cut-and-cover section extending out into the reservoir from the tunnel portal a sufficient distance to be clear of any possible sloughing of the banks. There are stop logs in the west face of this lower intake with crest 91 feet below the flow line which limit the reservoir draw-down, except that they may be

removed if necessary to an additional depth of 11 feet.

The view to the west from this point is across the broad flat reservoir site. Although this intake is 7 miles north of the Main Dam, about 400 of the total 415,000,000,000 gallons can be drawn here. Mount Pomeroy, about one mile to the west, will be an island in about the center of the reservoir. Mount Lizzie, an island about two miles to the southwest, is another prominent feature. Quabbin Dike closes a low point on the divide about 3 miles south of Mount Lizzie. Still other hills or islands may be seen to the northwest. The reservoir will extend even farther to the north, about 9 miles beyond the hills. Another branch of the reservoir, not visible from this point, will extend about 5 miles to the northeast. Still another branch of the reservoir, the West Branch, nearly 12 miles long, cannot be seen on account of the intervening Prescott peninsula (now in New Salem). The Main Dam is at the southern extremity of the West Branch. The total shore line will be over 175 miles, including over 100 islands. About 39 square miles will be flooded, 18 miles long and 150 feet maximum depth.

The entire area which the Commission has acquired for sanitary protection and construction purposes is about 80,000 acres, or more than double the area of Suffolk County, which includes Boston, Chelsea, Revere and Winthrop.

The service building provides garage space and quarters for a maintenance force; also a 10 k. v. a. gasoline-driven electric unit for furnishing lights and power to operate the crane which handles the screens and stop shutters in the shaft head house. The water supply is from a spring-fed concrete tank on the hillside. The main operating gates on the aqueduct will be located on the Wachusett end of the tunnel. The

flow in the aqueduct is temporarily controlled by a steel bulkhead, between this intake and Shaft No. 8, with massive tide gates which permitted the diversion from the Ware easterly to the Wachusett Reservoir while the westerly section of the tunnel was being constructed.

Discontinued Towns

Four towns of this reservoir area have been discontinued as such, and their areas have been divided and annexed to adjacent towns. This has been done for the former towns of Dana, Enfield, Greenwich and Prescott.

Quabbin Dike

Quabbin Dike, located at the southern end of the easterly part of the reservoir is completed.

The Dike is very similar in construction to the Main Dam, and nearly as large, since this point on the divide was very low, the pre-glacial north-south valley at this point being in fact larger than that to the west into which the river was diverted by the choking of glacial debris. The embankment is 2,140 feet long and 135 feet high above the valley from which a row of 31 caissons was sunk an additional depth of 129 feet to sound ledge to make a water-tight cut-off. The embankment contains 2,500,000 cubic yards. The roadway across the top of the Dike is 20 feet above the flow line. The riprap berm on the upstream face is 5 feet above the flow line and will take the pounding of waves at the south end of this 15-mile stretch of open water.

Quabbin Park Cemetery

Quabbin Park Cemetery is located about one mile south of the Dike, accessible from the main highway through

the town of Ware. More than 4,000 of the 7,500 bodies in the reservoir area have already been re-interred. This cemetery is laid out for 1,602 lots averaging eight graves each.

Main Dam

The party continued next along Route 9, one of the Commissions relocated circumferential highways, of which 36 miles have been built, passed on the right the entrance to the east access road, under construction to the Main Dam, and passed under the 66,000-volt transmission line of the New England Power Company, relocated around the south end of the reservoir for a distance of 8 miles. The route crossed Swift River and continued another mile up the hill and into the access road to the group of administration buildings now under construction.

The administration buildings are under construction at the westerly end of the Main Dam on the southerly shore of the West Branch of Quabbin Reservoir, with a 10-mile view up that branch. They will serve as general headquarters for maintenance of the Quabbin Reservoir and its watershed. The group consists of an office building, two residences, two garages for housing eighteen cars and trucks, an underground 60 foot by 70 foot hydroplane hangar, with connecting boathouse and general machine shop. The entire group is of fireproof construction protected by automatic sprinklers and heated by a steam plant in the basement of the office building equipped to burn either No. 5 oil or soft coal. Electric power is furnished by the Central Massachusetts Electric Company at 2,300 volts and transformed and distributed from an underground vault to this group and to the dam gate house. Auxiliary power for emergency operation is supplied from a 10 k. v. a. gasoline-driven gen-

erator set. The water supply is from a driven well with a 50,000-gallon storage tank on the hill to the southeast. The hydroplanes will, it is expected, be used for police patrol of the reservoirs and properties of the Metropolitan water system. Provision will be made for a short-wave radio receiving and broadcasting station.

A general view of the hydraulic fill operations at the Main Dam was obtained by walking a short distance easterly from the administration buildings. The river has been diverted through a tunnel under this west abutment 1,230 feet long and about 30 feet in diameter, which will later be plugged at a point directly beneath the control tower which extends from the tunnel up through the Dam and which will be topped by a gate structure on the upstream side of the roadway across the top of the Dam. The principal control works will be in a building to be built at the downstream end of the tunnel.

The party was conducted in groups. The Dam has already reached a height which entirely conceals the row of 38 caissons which were sunk from the bottom of the valley to sound ledge, a depth of 125 feet. However, the treatment of the ledge at the two abutments is in evidence. The belt conveyors with materials from the borrow pits converge at a point northeast of the Dam, where an inclined belt takes them to the top of the east hill to the mixing chamber where the water pumped from the pool on the Dam is added. The muck lines return to the Dam and deposit the mixed material along the upstream and downstream face. These pipe lines are generally rubber lined. The faces of the Dam are built up ahead of the beaches by drag-line scrapers which

select the coarsest material. The pool in which the finest impervious materials settle out is kept at an acceptable depth by the selection of proper borrow pit materials. The embankment when completed will be very similar to, but larger than, the Dike. It will be 2,640 feet long, 170 feet high above the river bed, and contain 4,000,000 cubic yards. The two steel pipe wells projecting above the pool will terminate at manholes alongside the finished highway over the Dam and permit inspection, through portholes, of the rate of consolidation of the core.

The principal nurseries developed by the Commission in its reforestation program are above the reservoir flow line about one mile north of the administration buildings. An extensive area is developed for seed beds and transplants.

The excursion was arranged through the courtesy of the Metropolitan District Commission, the Chairman of which is Mr. Eugene C. Hultman, and the Chief Engineer, Mr. Frank E. Winsor. Under the direction of Mr. Karl R. Kennison, Assistant Chief Engineer, the many details of the routing of the excursion party through the project and arranging for guides were carried out. The Excursion Committee consisted of Mr. Kennison; Mr. John P. Wentworth, chairman of the American Society of Civil Engineers, Northeastern Section; Mr. Samuel M. Ellsworth, Chairman of the Sanitary Section of the Boston Society of Civil Engineers; Mr. John H. Harding, Chairman of the Social Activities Committee; Mr. Edward S. Averell, who served as caterer for the lunches; and Everett N. Hutchins, Secretary of the Society.

BOSTON SOCIETY OF CIVIL ENGINEERS SCHOLARSHIP IN MEMORY OF DESMOND FITZGERALD AWARDED TO CHESLEY F. GARLAND, STUDENT AT NORTHEASTERN UNIVERSITY.

Chesley F. Garland of Belmont, Mass., a senior student, class of 1937, in civil engineering in the School of Engineering at Northeastern University, was awarded the Desmond FitzGerald Scholarship of the Boston Society of

Civil Engineers on May 4, 1938. The presentation of the scholarship of \$100 was made by Prof. Gordon M. Fair, Vice-President of the Boston Society of Civil Engineers, at a meeting of students of the school held at Jordan Hall.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

APRIL 27, 1938. — A regular meeting of the Boston Society of Civil Engineers was held this evening at Walker Memorial Building, Massachusetts Institute of Technology. This meeting was a joint meeting with the American Society of Civil Engineers, Northeastern Section and the Engineering Societies of New England.

The meeting was called to order by Karl R. Kennison, President, Boston Society of Civil Engineers. Eighty members and guests were present. Forty-eight persons attended the dinner preceding the meeting. Mr. John P. Wentworth, President of the Northeastern Section, American Society of Civil Engineers, and Frederick W. Bliss, President of the Engineering Societies of New England, officially represented these societies.

The President introduced the speaker of the evening, Mr. John C. Page, Commissioner of Reclamation, Department of the Interior, Washington, D. C., who gave a very interesting talk on "The National Conservation Program." A question period followed the talk. Sound-motion pictures portraying the work on the construction of Boulder Dam and the great projects carried on for the reclamation of the arid West were shown.

The meeting adjourned at 9.20 P.M.

EVERETT N. HUTCHINS, *Secretary*.

MAY 18, 1938. — A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club and was called to order at 7 P.M. by the President, Karl R. Kennison. One hundred and forty-five members and guests attended the meeting. Ninety-eight persons attended the supper.

Announcement was made of the election of the following new members on May 18, 1938:

Grade of Member: Alanson R. Curtis, Lawrence S. Burke,* William H. Fitzgerald,* James A. McCarthy,* John M. W. Sargent.*

Grade of Junior: Leslie W. Lenfest,† John N. Squarebriggs.†

The President announced that an all-day excursion is planned to the Quabbin Reservoir on June 11, details of which will be announced later.

The Chairman then introduced the speaker of the evening, Mr. Frank E. Winsor, chief engineer, Metropolitan District Water Supply Commission, who gave a very interesting talk on "Quabbin Reservoir — Recent Developments in the Construction of the Ware-Swift Water Supply for the Metropolitan District." The talk was illustrated by maps and motion pictures. A question period followed the talk. Also, the meeting was addressed by Mr. Eugene C. Hultman, Commissioner, Metropolitan District Water Supply Commission.

The meeting adjourned at 9.30 P.M.

EVERETT N. HUTCHINS, *Secretary*.

Sanitary Section

APRIL 12, 1938. — A special meeting of the Sanitary Section was held this evening at Harvard University. Preceding the meeting 21 members and guests gathered at the Harvard Faculty Club for dinner. The meeting was called to order by the Chairman, Samuel M. Ellsworth, in Room 110, Pierce Hall, at 7.30 P.M., with 39 members and guests present.

Prof. Gordon M. Fair introduced Dr. Charles E. Renn, Instructor in Sanitary Biology, Graduate School of Engineering, Harvard University, who described certain of the equipment used for instruction and research in sanitary engineering. The group then passed through the various laboratories where equipment for experimental work was described and operated.

The meeting adjourned at about 9.15 P.M.

RALPH M. SOULE, *Clerk*.

Highway Section

APRIL 6, 1938. — A regular meeting of the Highway Section of the Boston Society of Civil Engineers was held in the Society's rooms, 715 Tremont Temple, this evening. The meeting was called to order at 7.05 P.M., by the Chairman, Mr. Thomas C. Coleman.

The Chairman introduced the speaker of the evening, Mr. Joseph A. McCarthy, District Highway Engineer, Massachusetts Department of Public Works, who spoke on the subject "Modernizing the Newburyport Turnpike."

Mr. McCarthy told of the granting of a charter to the Turnpike Commission in 1803 and the completion of the toll road in 1805 at a total cost of about \$500,000. The turnpike became a free way in 1850 and maintenance was first started by the State in 1909.

A macadam surface was built from Malden to Essex Street, Saugus, in 1917, and the Turnpike was completed as a state highway 18 feet wide in 1922 at a total cost of \$1,273,000. It was again widened to 30 feet in 1930 at a cost of \$1,782,000.

In 1936 the present construction of double-barrelled highway contracts was undertaken; one section in Malden, Revere and Saugus, which included the Squire Road connections, was built about 2.93 miles in length at a cost of \$958,000, and another section 4.68 miles in Lynnfield and Saugus at a cost of \$1,500,000.

Another interesting note was that the vehicle traffic on week days amounts to 12,000 to 14,000, and 20,000 on Sundays.

The talk was illustrated by about 90 pictures, many of which were the popular "before and after" views.

The speaker was accorded a rising vote of thanks for his fine presentation.

The meeting adjourned at 8.30 P.M., with an attendance of 48 members.

ARTHUR E. HARDING, *Clerk*.

* Transfer from Grade of Junior.

† Transfer from Grade of Student.

APPLICATIONS FOR MEMBERSHIP

[July 20, 1938]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

CROUCH, CHARLES W. (Age 26, b. Gardner, Mass.) Graduated from Northeastern University, 1934, degree B.S. in civil engineering; October, 1931, through November, 1931, with E. W. Branch, Inc., Quincy, rodman in survey party; September, 1934, to September, 1935, with Florence Stove Company, Gardner, Mass., three months, stock clerk, nine months, mechanical drafting in engineering department, and drafting (with engineers of Frank D. Chase, Inc., Chicago) proposed plans for a new factory layout and additions to the plant; September, 1935, through June, 1937, teacher of business arithmetic, civics, general science, economics and business law, Gardner (Mass.) High School; July 12, 1937, to May 13 1938, engineering department, mechanical and structural drafting, including detail work in the design of high-pressure water-tube boilers, with Riley Stoker Corpora-

tion, Worcester, Mass. Refers to H. B. Alword, C. O. Baird, Jr., W. E. Butler, O. J. Calderara, A. E. Everett, Jr., E. A. Gramstorff.

For Transfer from Grade of Junior

HERSEY, HAROLD G., Hampton, N. H. (Age 31, b. Wolfeboro, N. H.) Graduated from Brewster Free Academy, Wolfeboro, N. H., June 27, 1927. Attended Northeastern University, September, 1927, to June, 1931, graduating with degree in C.E. Co-operative work while there: October 15 to November 17, 1928, with Carleton D. Reed, Woodsville, N. H., on bridge construction; December 26, 1928, to January 26, 1929, with Frasier Brace Construction Company, East Barnet, Vt., on dam construction, at Fifteen Mile Falls; March 6, 1929, to June 15, 1929, with H. F. Bryant & Son, as rodman on general engineering; June 17, 1929, to May 31, 1930, with Frasier Brace Company, on dam construction; June 9, 1930, to November 15, 1930, with Reed & Reed, Woolwich, Maine, as superintendent on bridge construction; December 22, 1930, to January 24, 1931, with New England Power Company, at Groveton, Pittsburg, N. H., as rodman on surveys, topographical, for new dam; May 11, 1931, to June, 1931, with Maine State Highway Department, Augusta, Me., on estimating; June, 1931 to 1933, with Maine State Highway Commission, bridge division, as a resident engineer on bridge construction; 1933 to date, with New Hampshire Highway department, as follows: 1 year, transitman on surveys for Federal Aid Roads; two years, resident engineer on bridge construction, concrete, stone, steel, etc.; two years, in Division Office as transitman, drafting, surveys, general maintenance work, etc. Refers to H. B. Alword, C. O. Baird, Jr., C. S. Ell, E. A. Gramstorff, R. S. Harmon.

Transfer from Grade of Student

BERGSTROM, RICHARD G., Lynn, Mass. (Age 26, b. Lynn, Mass.) Graduated from Northeastern University, 1937; during co-operative periods, as follows: with Joseph Bergstrom, Contractor, Lynn,

Mass., 1934 to 1936; with Whitman & Howard, Civil Engineers, Boston, September, 1936, to September, 1937; General Electric Company, River Works, Lynn, Mass., September, 1937, to January, 1938; at present with Everett M. Brooks, Civil Engineer, Newtonville. Have been building and surveying, 1935 to date, for self. Refers to *H. B. Alvord*, *C. O. Baird, Jr.*, *A. E. Everett, Jr.*, *E. A. Gramstorff*.

PUTZEL, DANIEL J., JR., Hamden, Conn. (Age 21, b. New Haven, Conn.) Graduated from New Haven High School, 1934; graduated from Northeastern University, June, 1938, degree B.S. in civil engineering; for past two years with New York, New Haven & Hartford Railroad during co-operative working periods; and now in maintenance department at Hartford,

Conn. Refers to *H. B. Alvord*, *C. O. Baird, Jr.*, *C. S. Ell*, *E. A. Gramstorff*, *L. W. Lenfest*.

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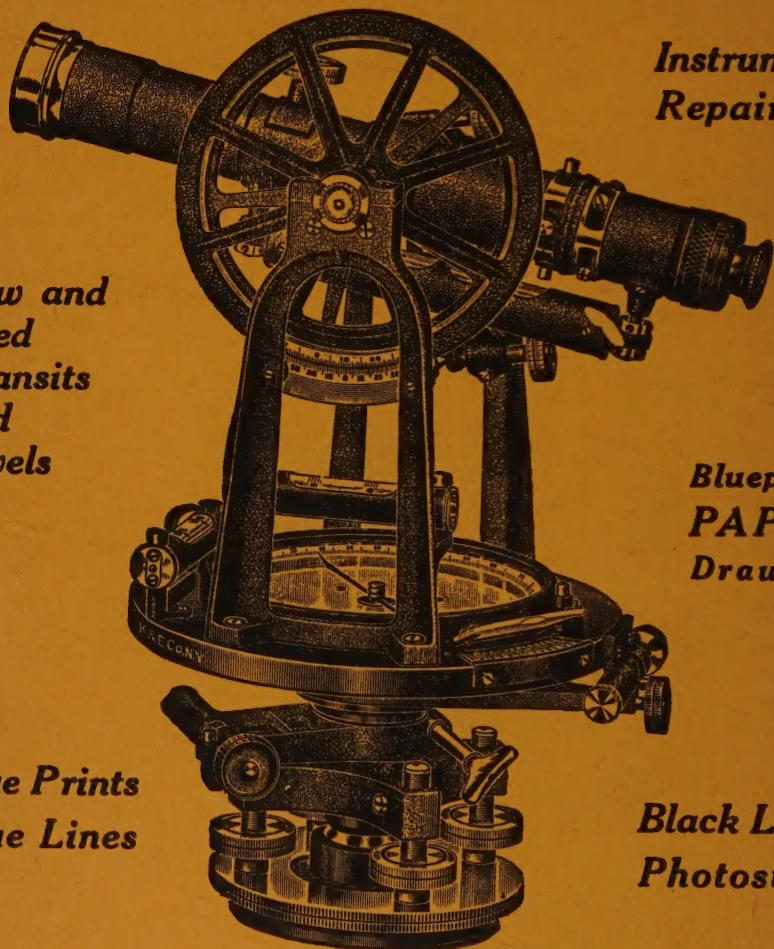
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